INCREMENTAL DYNAMIC ANALYSIS APPLIED TO SEISMIC RISK ASSESSMENT OF BRIDGES

J. B. Mander[1], R. P. Dhakal[2] and N. Mashiko[3]

ABTRACT

Incremental Dynamic Analysis (IDA) is applied in a Performance-Based Earthquake Engineering context to investigate expected structural response and damage outcomes to highway bridges. This quantitative risk analysis procedure consists of: adopting a suitable suite of ground motions and performing IDA on a nonlinear model of the prototype structure; summarize and parameterize the IDA results into various percentile performance bounds; and integrate the results with respect to hazard intensity-recurrence relations into a probabilistic risk format. An illustrative example of the procedure is given for reinforced concrete highway bridge piers, designed to New Zealand, Japan and Caltrans specifications. It is shown that for bridges designed to a "Design Basis Earthquake" that has a 10 percent probability in 50 years with PGA = 0.4g, and detailed according to the specification of each country, should perform well without extensive damage. However, if a larger earthquake occurs, such as a maximum considered event which has a probability of 2 percent in 50 years, then extensive damage with the possibility of collapse may be expected.

INTRODUCTION

Performance Based Earthquake Engineering (PBEE) procedures require the prediction of the seismic capacity of structures which is then compared to the local seismic demand. The interrelationship between the two gives an inference of the expected level of damage for a given level of ground shaking. In order to estimate structural performance under seismic loads, Vamvatsikos and Cornell (2004) proposed a computational-based methodology called "Incremental Dynamic Analysis (IDA)". The IDA approach is a new methodology which can give a clear indication of the relationship between the seismic capacity and the demand. With respect to seismological *intensity measures (IM)*, such as peak ground acceleration, engineers can estimate principal response quantities in terms of governing *engineering demand parameters (EDP)*, such as the maximum deflection or drift of the structure.

[1]Professor, Dept. of Civil Engineering, University of Canterbury, Private Bag 4800, Christchurch, NZ
[2]Senior Lecturer, Dept. of Civil Engineering, University of Canterbury, Private Bag 4800, Christchurch, NZ
[3]Graduate Student, Dept. of Civil Engineering, University of Canterbury, Private Bag 4800, Christchurch, NZ

The IDA approach involves performing nonlinear dynamic analyses of a prototype structural system under a suite of ground motion records, each scaled to several *IM* levels designed to force the structure all the way from elastic response to final global dynamic instability (collapse). From IDA curves, limit states can be defined. The probability of exceeding a specified limit state for a given *IM* (eg PGA) can also be found. The final results of IDA are thus in a suitable format to be conveniently integrated with a conventional seismic hazard curve in order to calculate mean annual frequency of exceeding a certain damage limit-state capacity.

This paper develops the IDA process specifically for bridge structures. What is new here is the way in which IDA results are quantitatively modelled and then integrated into a probabilistic risk analysis procedure whereby the seismic intensity-recurrence relationship (the seismic demand) is viewed with respect to the damage propensity of a specific bridge structure (structural capacity). Confidence intervals and damage outcomes for given hazard intensity levels, such as the Design Basis Earthquake (DBE) or a Maximum Considered Earthquake (MCE) earthquake, can be evaluated.

IDA-BASED SEISMIC RISK ASSESSMENT

Step 1: Select ground motion records and Hazard–Recurrence Risk Relation

In order to perform IDA, a suite of ground motion records are needed. In their previous study, Vamvatsikos and Cornell (2004) used 20 ground motion records to analyse mid-rise buildings in order to provide sufficient accuracy of seismic demands. The same ground motions used by Vamvatsikos and Cornell (2004) were adopted for this study. These earthquakes have Richter magnitudes in the range of 6.5-6.9 with moderate epi-central distances mostly in the range of 16 to 32 km; all these ground motions were recorded on firm soil. Figure 1 (a) shows response spectra for each of the 20 earthquake ground motions scaled to the same *IM* that is a PGA of 0.4g. A significant degree of variability is evident with respect to the median spectral curve. Figure 1 (a) also presents a plot of the lognormal coefficient of variation (β_D), sometimes referred to as the dispersion, across the spectrum. Due to the consistent and relatively low values of β_D for periods up to 1.6 seconds, it is evident that PGA serves (for this suite of earthquakes) as an appropriate *IM*.

An annual frequency-dependant scale factor λ_T such that $S_a^{(T=Tr)}=\lambda S_a^{(T=475yrs)}$ is required to scale spectral magnitudes (the *IM*), with respect to the reference return period of 475 years. Values for the return period factor have been derived by drawing a representative line through the hazard curves (PGA as a function of annual frequency or return period) as illustrated in Figure 1 (b). The relationship is given by:

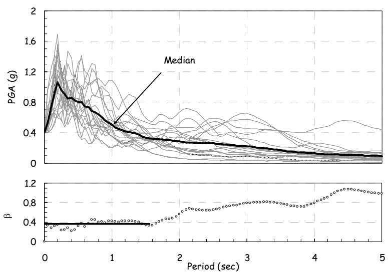
$$S_a^{(T=Tr)} = \lambda_T S_a^{(T=475)} = S_a^{(T=475)}\left(\frac{Tr}{475}\right)^q = \frac{S_a^{(T=475)}}{(475p_a)^q} \qquad (1)$$

in which $S_a^{(T=Tr)}$ = PGA relevant to its return period ; $S_a^{(T=475)}$ = PGA at a return period of 475 years (10 percent probability in 50 years); Tr = return period; p_a = annual frequency (p_a=1/Tr); and q = an exponent based on local seismic hazard-recurrence relations. According to the data specified in design codes, values of q for New Zealand, Japan and Caltrans designs are determined to be 0.333, 0.418 and 0.29, respectively.

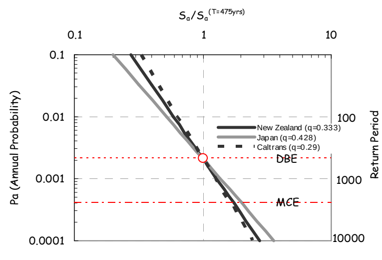
Step 2: Perform Incremental Dynamic Analysis

Once the model and the ground motion records have been chosen, IDA is performed. Thus a nonlinear computational model of the prototype structural system should be developed. To start the analysis, the chosen earthquake records need to be scaled from a low *IM* to several higher *IM* levels until structural collapse occurs.

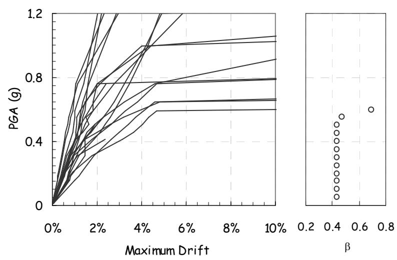
For each increment of *IM*, a nonlinear dynamic time history analysis is performed. Analyses are repeated for higher *IM*'s until structural collapse occurs. Locating the maximum drift observed in an analysis gives one point in the *IM* vs. *EDP* (PGA vs. drift) domain. As shown in Figure 1 (c), connecting such points obtained from all the analyses using each earthquake record with different *IM*'s gives the IDA curves for all earthquakes in the suite. It may also be of interest to analyse the variability of the response outcomes for a given level of *IM*. Results typically show a lognormal distribution of drift (displacement) outcomes with the dispersion factor, β, plotted on the right side of Figure 1 (c).



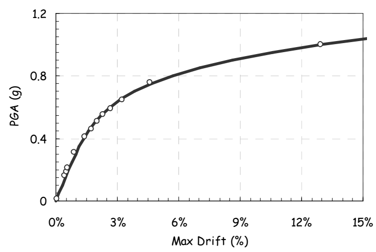
(a) Step 1a: Ground motion records scaled to a common IM of PGA = 0.4g



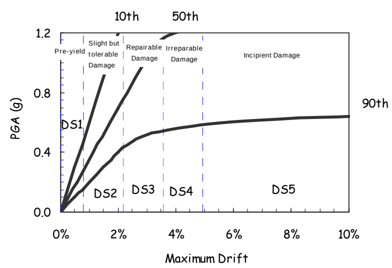
(b) Step 1b: Hazard–Recurrence Relation



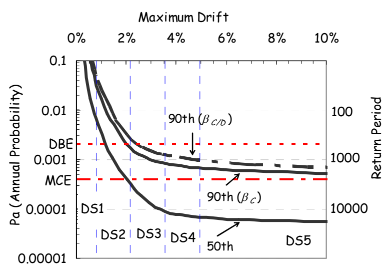
(c) Step 2: Conduct IDA



(d) Step 3 Model IDA results with the R-O relationship



(e) Step 4: Assign Damages States



(f) Step 5: Risk modelling accounting for uncertainty

Figure 1. The Steps in conducting an IDA-Based Seismic Risk Assessment

Step 3: Model the IDA curve and statistical outcomes

In their previous study, Vamvatsikos and Cornell (2004) modeled their IDA curves by using multiple interpolation spline functions. It is considered that such an approximation is cumbersome and not particularly useful for subsequent risk analysis. Therefore, in this study several single functional relations were explored, and the Ramberg-Osgood equation (R-O) was adopted as the most suitable. The R-O equation can be written in the following two forms:

$$\frac{\theta}{\theta_c} = \frac{S_a}{S_c} + \left(\frac{S_a}{S_c}\right)^r = \frac{S_a}{K\theta_c}\left(1 + \left|\frac{S_a}{S_c}\right|^{r-1}\right) \tag{2}$$

in which θ = drift; K = the initial slope of the IDA curve in the proportional range; S_c = "critical" earthquake acceleration that occurs at the onset of large drifts that subsequently lead to collapse; S_a = earthquake acceleration (in this study PGA is used); $\theta_c = S_c / K$ is a "critical" drift; and r = constant.

In Equation (2) the three control parameters (S_c, r, and either θ_c or K) are estimated using nonlinear least squares analysis for each individual earthquake ground motion IDA data set. Figure 1 (d) illustrates the fit between the IDA data points and the fitted R-O curve for one specific case.

Although the values for each of the control parameters for each of the IDA curves are different, they can then be examined collectively and a statistical analysis on the parameters can be performed. Studies show that the parameters are lognormally distributed. Therefore by ascertaining median values of each parameter the 50th percentile IDA response can be represented by an individual R-O median curve. Likewise by examining variability of individual IDA distributions, parameters that represent curves of other bounds of interest, such as the 10th and 90th percentiles may be found. Figure 1 (e) illustrates fitted IDA curves for the 10th, 50th and 90th percentile response demand.

Step 4: Assign damage limit states

Once the three (10th, 50th and 90th percentile) lines have been generated, it is possible to determine the expected drift for an earthquake with a certain level of intensity. Emerging international best practice for seismic design is tending to adopt a dual level intensity approach, that is: (i) a DBE represented by a 10% in 50 years ground motion; and (ii) a MCE represented by a 2% in 50 years earthquake.

Several damage limit-states can be defined on the IDA curves developed. In their previous research, Vamvatsikos and Cornell (2004) applied building use criteria of Immediate Occupancy (IO) and Collapse Prevention (CP) limit-states to their IDA curves based on building use criteria. In this study, the definitions of damage limit states were extended by adopting Mander and Basoz (1999) definitions of damage states for bridges, as listed in Table 1, with the result of assigning damage states to the IDA fractile curves illustrated in Figure 1 (e).

Table 1. Damage States adapted from HAZUS (1999)

	Damage State	*Failure Mechanism*	*Repair required*	*Outage*
DS1	None	Pre-Yielding	None	None
DS2	Minor/Slight	Post-Yielding Minor spalling	Inspect, Adjust, Patch	< 3 days
DS3	Moderate	Post Spalling, Bar buckling	Repair components	< 3 weeks
DS4	Major/Extensive	Degrading of strength, Bar fracture	Rebuild components	< 3 months
DS5	Complete/Collapse	Collapse	Rebuild structure	> 3 months

The first and last damage states can be easily defined: DS=1 represents mostly elastic behaviour, it therefore concludes at the onset of damage which is most easily defined at the yield drift (displacement) of the structure; and DS=5 commences at the onset of collapse and as described above this is best defined when $\theta > 2\theta_c$.

The other damage stages (DS=2, 3, and 4) are more subjective in their definitions. It is suggested that the boundary separating DS=3 and DS=4 be defined at that level of drift where the structure would be deemed to have suffered irreparable damage such that the structure would likely be abandoned. This may be evidenced by: (i) excessive permanent drift at the end of the earthquake; (ii) severe damage to critical elements such as buckling of longitudinal reinforcing bars or the fracture of transverse hoops and/or longitudinal reinforcing bars.

The boundary separating DS=2 and DS=3 should be defined as that level of damage that would necessitate repairs that need to be undertaken. Such repairs lead to temporary loss of functionality. For reinforced concrete bridge substructures, this usually occurs when spalling of cover concrete is evident. This displacement can also be found by analysis when the cover concrete compression strain exceeds the spalling strain at say ε_{spall}=0.008. At drifts below this boundary (i.e., DS=2) damage is considered to be slight and tolerable.

Step 5: Risk modelling and accounting for uncertainty

From the IDA curves developed, the curves can be modified more elegantly by substituting hazard curves based on Equation (1) into Equation (2) to give:

$$\frac{\theta}{\theta_c} = \frac{S_a^{(T=475)}}{S_c(475 p_a)^q} + \left(\frac{S_a^{(T=475)}}{S_c(475 p_a)^q} \right)^r \tag{3}$$

Note that the parameters S_c, θ_c, and r are dependent on confidence interval.

In the foregoing analysis it must be emphasized that the resulting variability in response results entirely from the randomness of the input motion that is the seismic demand. This is because the computational modeling is conducted using crisp input data. However, the structural resistance both in terms of strength and displacement capacity is also inherently variable. Moreover, the computational modelling, although it may be sophisticated, is not exact; there is a measure of uncertainty that exists between the predicted and the observed response.

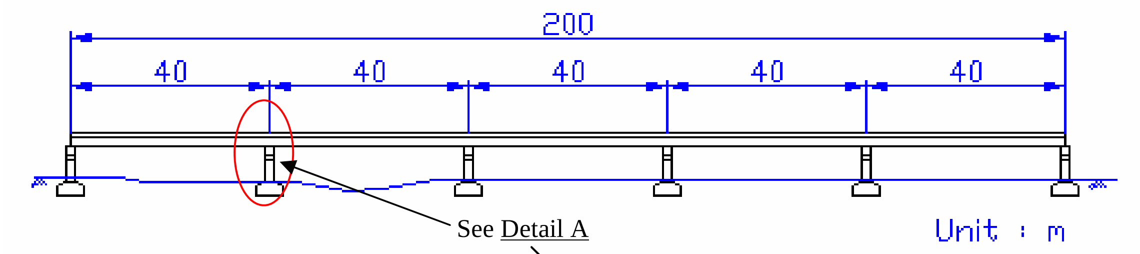
To encompass the randomness of seismic demand along with the inherent randomness of the structural capacity and the uncertainty due to inexactness of the computational modelling it is necessary to use an integrated approach as suggested by Kennedy et al (1980). The composite dispersion value for the lognormal distribution can be expressed as

$$\beta_{C/D} = \sqrt{\beta_C^{\ 2} + \beta_D^{\ 2} + \beta_U^{\ 2}} \tag{4}$$

in which β_D = coefficient of variation for the seismic demand which arises from record-to-record randomness in the earthquake ground motion suite; β_C = coefficient of variation for the capacity which arises as a result of the randomness of the material properties that affect strength, in the case of reinforced concrete bridge columns this is due to randomness in the steel yield strength and assumed to be β_C = 0.2 in this study; and β_U = lognormal dispersion parameter for modelling uncertainty which is assumed to be β_U = 0.25 in this study. The hazard recurrence curves that include both aleotory and epistemic uncertainty can be seen plotted as the dotted 90th percentile line for $\beta_{C/D} = 0.5$ in Figure 1 (f).

CASE STUDY OF BRIDGE PIERS

Results of a comparative study on three bridge piers, initially designed by Tanabe (1999), is presented herein. The three piers were designed using governing specifications of NZ, Japan and USA (Caltrans). All three piers are 7m high and were taken from a "long" multi-span highway bridge on firm soil with a 40m longitudinal span and a 10m transverse width. The weight of the super-structure at each pier is assumed to be 7 MN. Elevation views of the whole bridge and piers together with the design parameters for the three piers are given in Figure 2.



	N Z	Japan	Caltrans
D	1700	2000	2000
D'	1540	1834	1838
PHZ	1700	4000	3000
$P/A_g f'_c$	0.15	0.11	0.11
Bar	28-D32	28-D51	32-D41
ρ_t	0.99%	1.82%	1.34%
Spiral	R20@170	R20@115	R20@85
ρ_s	0.49%	0.61%	0.78%

ρ_s = the volume ratio of the volume of the spiral to the volume of the confined concrete

ρ_t = the ratio of the longitudinal bars area to the pier's cross sectional area

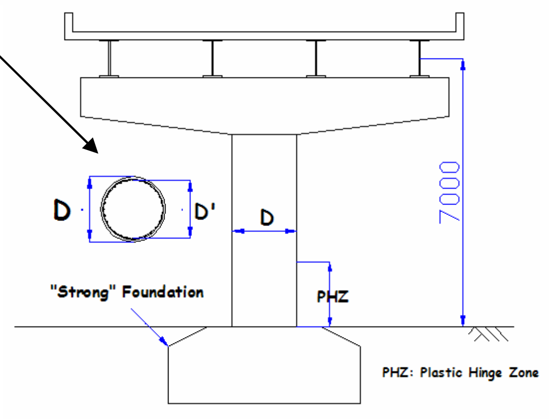


Figure 2 The Prototype Bridge and Pier Details.

IDA Procedures

Dynamic time history inelastic analyses were carried out for the 20 selected earthquake records using a nonlinear structural analysis program RUAUMOKO [Carr; 2003]. But prior to performing the IDA, pushover analyses were conducted to enable a single-degree of freedom model for each of the three reinforced concrete circular bridge piers to be established. A modified Takeda rule [Carr; 2003] was adopted to model the hysteretic performance of the piers. Figure 3 (a) presents the data obtained from the IDA computational investigation which are plotted along with their respective dispersions for the three piers. Table 2 presents the 20 earthquakes used in the IDA, along with the parameters obtained to fit the set of IDA results to the R-O relationship given in Equation 2. Fitted IDA curves for the 10th, 50th and 90th percentile bands are shown for each bridge pier in Figure 3 (b). Also shown in Figure 3 (b) are the five damage state bands described above and listed in Table 1.

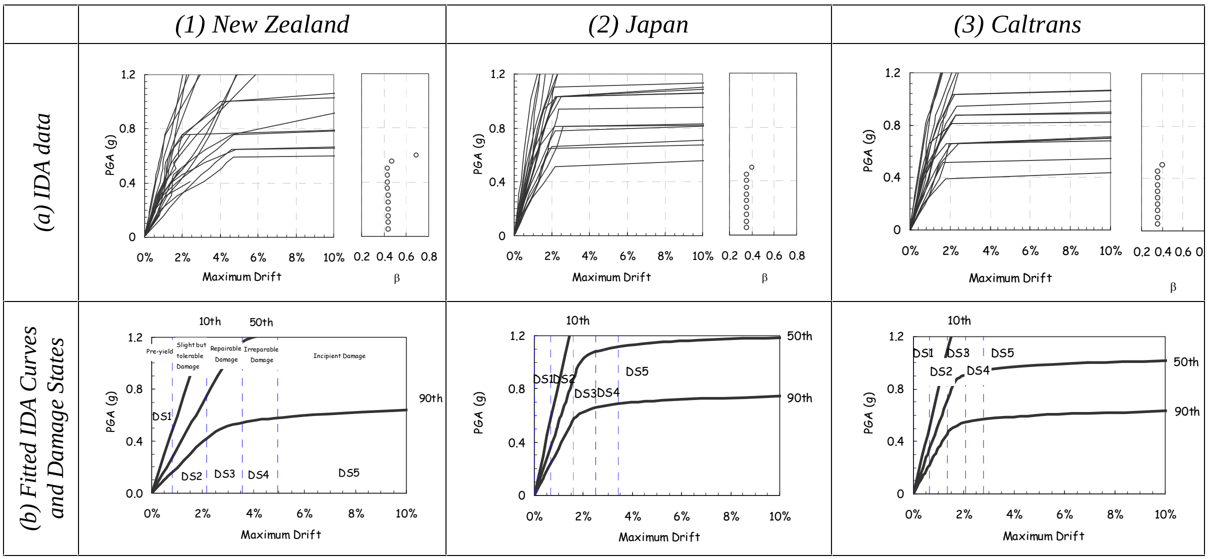


Figure 3 IDA Results for New Zealand, Japan and Caltrans Bridge pier

Table 2. Earthquakes use in IDA with the results of the R-O modeling and Statistical Parameter Identification

				New Zealand			Japan			Caltrans		
EQ No	Event	Year	PGA g	S_c g	θ_c %	r	S_c g	θ_c %	r	S_c g	θ_c %	r
1	Loma Prieta	1989	0.159	0.80	2.1%	25	1.05	2.1%	24	0.86	1.7%	28
2	Imperial Valley	1979	0.057	1.80	4.3%	15	1.50	2.1%	26	1.20	1.7%	25
3	Loma Prieta	1989	0.279	1.05	4.2%	28	0.70	2.0%	32	0.66	2.0%	27
4	Loma Prieta	1989	0.244	1.80	4.0%	18	1.30	2.0%	29	1.22	2.1%	32
5	Loma Prieta	1989	0.179	1.60	6.7%	15	1.10	2.2%	19	0.95	1.9%	43
6	Imperial Valley	1979	0.309	1.20	4.4%	34	1.05	2.1%	30	0.90	2.1%	42
7	Loma Prieta	1989	0.207	0.75	2.3%	5	0.95	2.3%	24	0.82	2.1%	45
8	Imperial Valley	1979	0.117	0.78	2.0%	36	1.30	2.2%	27	1.05	1.9%	43
9	Imperial Valley	1979	0.074	0.60	2.0%	9	0.80	1.6%	32	0.65	1.2%	32
10	Loma Prieta	1989	0.371	0.70	4.7%	20	0.53	2.0%	28	0.38	1.3%	15
11	Loma Prieta	1989	0.209	0.78	3.7%	25	1.02	1.7%	34	0.80	1.2%	36
12	Superstition Hills	1987	0.180	0.60	2.9%	15	0.80	1.5%	14	0.65	1.3%	25
13	Imperial Valley	1979	0.254	1.20	4.8%	24	0.80	2.3%	28	0.66	1.7%	21
14	Imperial Valley	1979	0.139	1.40	2.5%	18	1.70	1.9%	31	1.60	1.9%	25
15	Imperial Valley	1979	0.110	1.00	2.6%	19	1.10	1.9%	35	1.05	1.9%	36
16	Loma Prieta	1989	0.370	3.50	5.6%	18	1.40	2.5%	31	1.24	2.4%	21
17	Superstition Hills	1987	0.200	0.60	4.0%	25	0.65	1.6%	41	0.52	1.4%	34
18	Imperial Valley	1979	0.042	2.10	3.8%	11	2.10	2.4%	21	1.80	2.3%	8
19	Loma Prieta	1989	0.269	1.05	4.6%	19	0.84	2.6%	24	0.70	2.4%	30
20	Loma Prieta	1989	0.638	3.10	5.0%	35	1.95	2.2%	17	1.60	1.9%	12
			10^{th}	2.90	5.9%	34.8	1.83	2.5%	37.1	1.60	2.4%	46.0
			50^{th} [median]	1.32	3.8%	20.7	1.13	2.1%	27.4	0.97	1.8%	29.0
			90^{th}	0.60	2.5%	12.3	0.70	1.7%	20.2	0.58	1.4%	18.3
			β	0.61	0.34	0.41	0.38	0.15	0.24	0.40	0.21	0.36

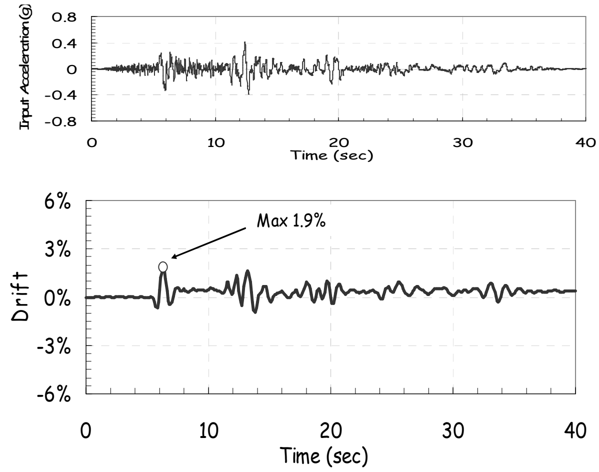
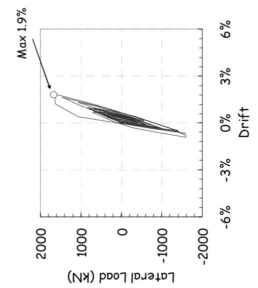
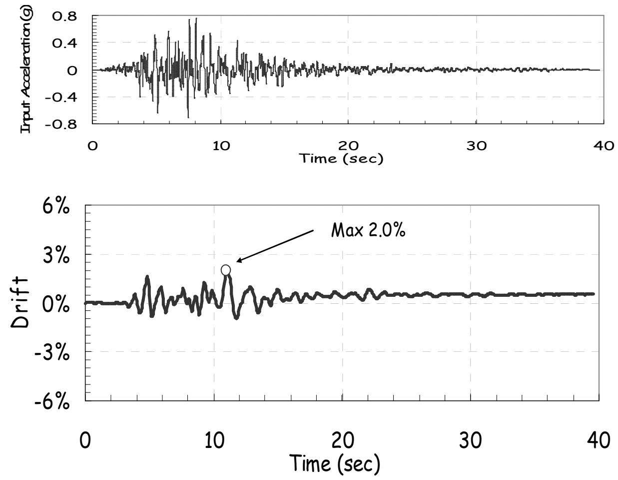
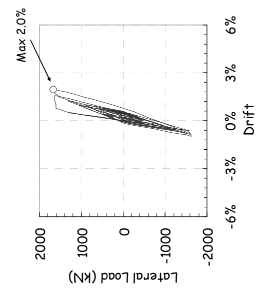
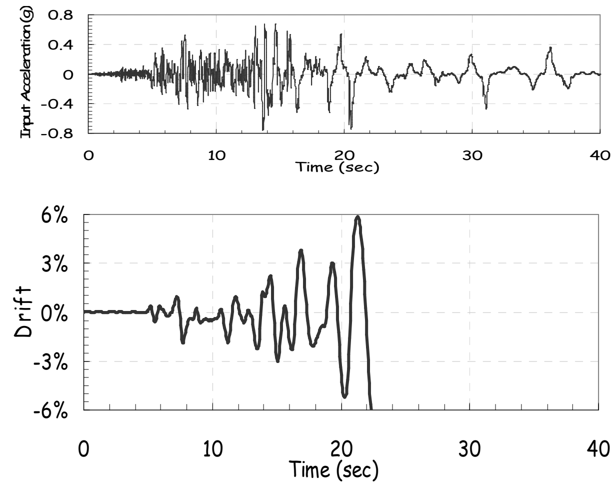
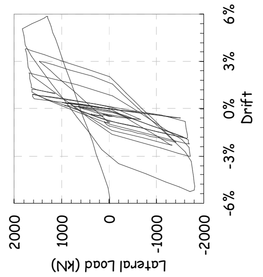
	Time histories of input accelerations and drift response	*Lateral Load vs. Drift response*
EQ13: DBE (90^{th} percentile)		
EQ4: MCE (50^{th} percentile)		
EQ17: MCE (90^{th} percentile)		

Figure 4. Example Dynamic Analyses for the New Zealand Bridge Pier

From the IDA results it is possible to identify those earthquakes that, in a probabilistic sense inflict the most damage. It is, therefore, of interest to scrutinize the structural performance for a few selected ground motions that correspond to the desired performance bounds. Figure 4 illustrates this process for the New Zealand designed bridge pier. Earthquake 13 is selected firstly from Table 2. This was found to represent a 90th percentile performance of the DBE—that is 10% in 50 year motion with an *IM* of PGA = 0.4g. From the results presented in Figure 4 (a) it is evident that the performance expectation results in only "slight" damage (i.e. DS2). Thus a high degree of confidence can be placed on achieving satisfactory performance under DBE-like motions. Similar conclusions can be drawn for both the Japanese and Caltrans designs.

Under MCE—that is *IM = 0.8g* with a 2% probability in 50 year—two earthquakes, 4 and 17 in Table 2, have respectively been selected to represent the median (50th percentile) expected response and 90th percentile response for the New Zealand bridge pier. It is of interest to examine the results in Figure 4 (b) and (c), respectively. These show dramatically different levels of behaviour from "slight" damage (DS2), to "complete" damage or toppling (DS5). This wide variability of response needs to be better understood.

Hazard-recurrence risk assessment

The diversity in seismic behavior outcomes can be better understood by examining the responses in a probabilistic risk sense. By transforming the IDA curves using Equation (1), hazard recurrence-drift-damage interrelationships can be obtained. This result is shown with the solid lines in Figure 5 (a). Note that these curves only display uncertainty due to the record-to-record randomness inherent in the suite of ground motions used in the IDA. To include all sources of uncertainty, Equation (4) with $\beta_{C/D} = 0.5$ is applied to the 90th percentile curves, as shown by the dashed lines in Figure 5 (a). Although this does not affect the expected outcome for the DBE level of shaking, it can markedly affect the degree of damage expected for the MCE. Note that when $\beta_{C/D}$ is used the response curves are asymptotic in the vicinity of the MCE.

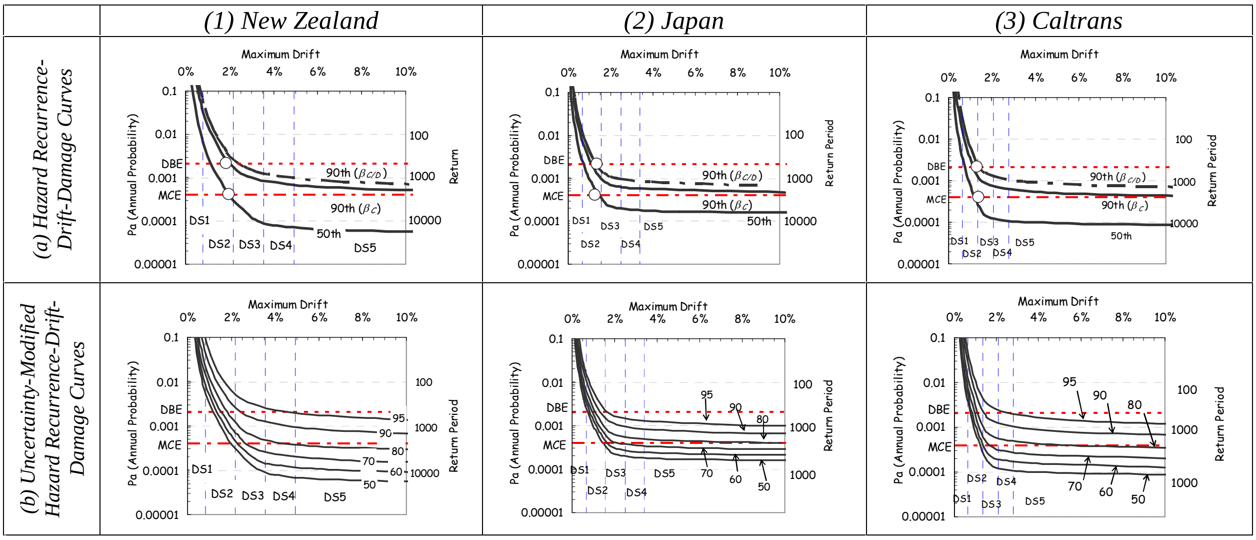


Figure 5. IDA Results for New Zealand, Japan and Caltrans Bridge pier

Equation (3) along with the IDA parameters for the 50th percentile (median) values and composite $\beta_{C/D}$ values as given by Equation (4) and Table 2, can be used to plot a full set of more detailed hazard recurrence-drift-damage curves. Figure 5 (b) presents 95th, 90th, 80th, 70th, 60th and 50th percentile curves for the three bridges. From this quantitative risk analysis it is evident that for each of the three bridge designs, one can be 95 percent confident of survival without collapse for a DBE with a 10 percent probability in 50 years. For a rarer event, such as an MCE that has a 2 percent probability of recurrence in 50 years, it is evident that one's confidence in the performance is substantially reduced. For the New Zealand and Caltrans designs, one can only be some 80 percent confident that the bridge will survive without collapse—this implies there is a 20% chance of collapse. Moreover, for each design there is roughly a 25 percent chance that irreparable damage will occur.

CONCLUSIONS

This paper has presented a study based on using IDA in the context of a quantitative seismic risk assessment. The following conclusions are drawn:

1. It is important to analyse bridge structures under high level of shaking as large displacements can occur that can lead to structural collapse. The IDA approach is a systematic method for achieving this end. It is possible to parameterise the outcomes using the Ramberg-Osgood (R-O) function. Statistical analysis of the control parameters in the R-O equation gives a good indication of the level of shaking needed to cause collapse.

2. A seismic risk analysis can be developed when IDA is combined with site-dependent hazard-recurrence relations and compiled with damage indices. In this way, risk can be posed as the probability of the hazard times the consequential outcome for a given level shaking in terms of structural damage for a level of confidence in that outcome.

3. A bridge designed to a "Design Basis Earthquake" that has a 10 percent probability in 50 years with PGA = 0.4g, and detailed according to the specification of each country, should perform well without extensive damage. However, if a larger earthquake occurs, such as a "maximum considered event" which has a probability of 2 percent in 50 years, then extensive damage with the possibility of collapse may be expected.

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